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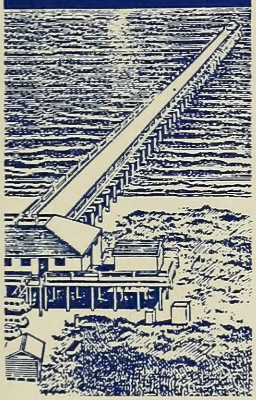

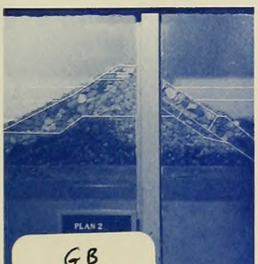

TECHNICAL REPORT CERC-90-7

KING HARBOR, REDONDO BEACH, CALIFORNIA, BREAKWATER STABILITY STUDY

Hydraulic Model Investigation

by

Ernest R. Smith, Willie G. Dubose, Robert D. Carver

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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A hydraulic model investigation of proposed breakwater modifications to King Harbor, Redondo Beach, California, was conducted at a geometrically undistorted scale of 1:35, model to prototype. Five breakwater cross sections were each subjected to 15 design wave and water-level conditions. Plans 1 through 3 consisted of variations of armor unit size on the rehabilitation sections. A transition layer of small stones was included in Plans 4 and 5 to reduce wave transmission. Data collected were displaced stones, qualitative overtopping observations, and wave transmission. Plan 5 gave the best combination of stability and reduction of wave transmission.

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PREFACE

The model investigation reported herein was requested by the US Army Engineer District, Los Angeles (SPL), and conducted at the Coastal Engineering Research Center (CERC) of the US Army Engineer Waterways Experiment Station (WES). Authorization for WES to perform the study was subsequently granted by SPL in SPL Intra-Army Order E86890016 dated 17 November 1988.

Model testing was conducted at WES from March to August 1989 under the general direction of Dr. James R. Houston, Chief, CERC; Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC; Mr. C. E. Chatham, Jr., Chief, Wave Dynamics Division (WDD); and Mr. D. D. Davidson, Chief, Wave Research Branch (WRB). Tests were conducted by Mr. Ernest R. Smith, Wave Processes Branch (WPB); Mr. Willie G. Dubose, WRB; Mr Robert D. Carver, WRB; Ms. Brenda J. Wright, WRB; Mr. Lonnie L. Friar, Instrumentation Services Division (ISD), and Mr. Richard H. Floyd, ISD. This report was prepared by Messrs. Smith, Dubose, and Carver. Ms. Lee T. Byrne, Information Technology Laboratory, edited this report.

Liaison was maintained during the course of the investigation with SPL by means of conferences, progress reports, and telephone conversations. Points of contact with SPL were Messrs. Arthur T. Shak and Chuck Mesa.

COL Larry B. Fulton, EN, was Commander and Director of WES during report publication. Dr. Robert W. Whalin was Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2,000 pounds, mass)	907.1847	kilograms

KING HARBOR, REDONDO BEACH, CALIFORNIA, BREAKWATER STABILITY STUDY

Hydraulic Model Investigation

PART I: INTRODUCTION

Prototype

1. King Harbor is a man-made harbor located on the Pacific coast in the City of Redondo Beach, California, at the southern end of Santa Monica Bay (Figure 1). The harbor is approximately 17 miles* southwest of central Los Angeles and serves as a port of call designed to accommodate small craft. Two rubble-mound breakwaters, north and south, provide protection for the harbor, which includes three basins enclosed by four moles with reveted slopes.

2. The principal economy of the harbor includes commercial and recreational fishing and pleasure boating. Attraction of the recreational facilities results in a healthy economy for adjoining businesses.

Existing Breakwaters

3. The North Breakwater (NB) was constructed during 1937-1939 by the Public Works Administration to a length of 2,370 ft with a crest elevation of +10 ft mean lower low water (mllw). The crest elevation was raised to +14 ft mllw, and the breakwater was extended to 5,200 ft in 1958. The 600-ft-long South Breakwater was also constructed in 1958 by the US Army Corps of Engineers, at a crest elevation of +14 ft mllw. An 8-ft-high seawall, 1,020 ft long, was added to the northern end of the NB in 1962 by the City of Redondo Beach. Additionally, the crest elevation of the NB was raised to +20 ft mllw from Sta 15+50 to 36+00 during 1964.

4. The NB has experienced severe damage during winter storms when high waves and water levels have combined. Repairs have been required after storm

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

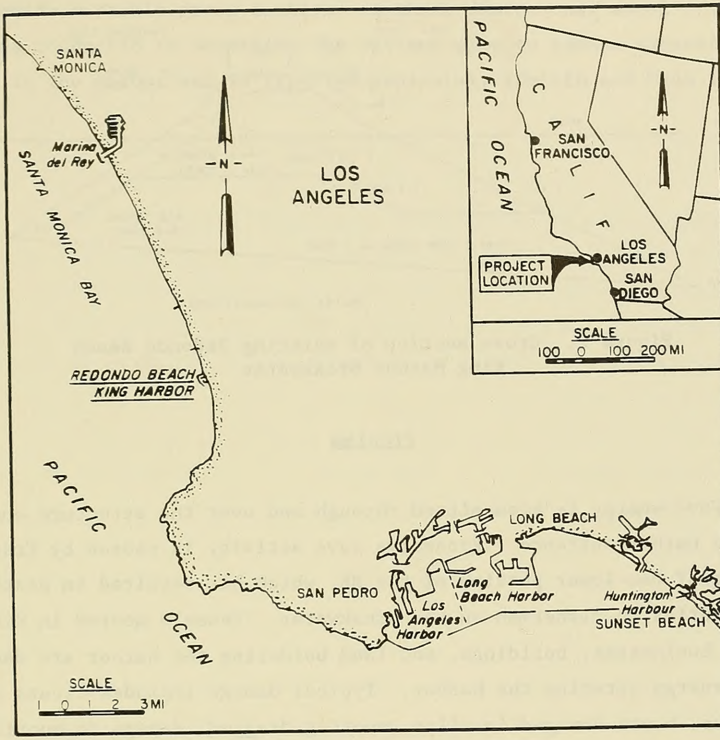

Figure 1. Location map, King Harbor, Redondo Beach, California

damage in 1960, 1982, 1983, 1986, and 1988. A thorough history of construction and repair work to the NB can be found in Hales (1987).

5. A condition survey of the NB, performed in 1988, showed the crest elevation was +12 ft mllw from Sta 36+00 to 52+00. The breakwater was repaired, and the present cross section from Sta 36+00 to 52+00 is shown in Figure 2. Base and core stone were less than 2 tons. The inner breakwater and sea side of the breakwater consisted of 1- to 13-ton stone. The crest elevation was +16 ft mllw with crown stone ranging from 11 to 19 tons ("A" Stone). The harbor side armor stone between +5 ft mllw and -10 ft mllw was 6 to 11 tons ("A-1" Stone). Armor stone below -10 ft mllw on the harbor side was 3 to 8 tons ("A-2" Stone). The "A" Stone, "A-1" Stone, and "A-2" Stone were added during breakwater emergency repairs in 1988.

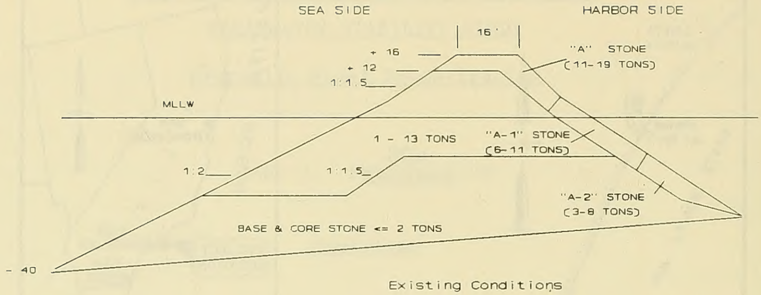

Figure 2. Cross section of existing Redondo Beach King Harbor breakwater

Problem

6. Wave energy is transmitted through and over the structure and also through the harbor entrance. Excessive wave activity is caused by frequent overtopping of the lower portion of the NB, which has resulted in death and serious injuries to fishermen on the breakwater. Vessels moored in King Harbor and businesses, buildings, and land bordering the harbor are damaged from wave energy entering the harbor. Typical damage includes cleats pulled out of docks, boats damaged in slips, mooring dragged, damage to mooring lines, city-owned waterfront property flooded, destruction of property due to wave forces, and small boats capsized.

Purpose of Study

7. Local interests have expressed a need for modifications to the breakwaters to eliminate excessive wave action in the harbor and damages to the surrounding businesses. Plans for new businesses and beautification projects are being considered upon completion of the modifications.

8. At the request of the US Army Engineer District, Los Angeles (SPL), a coastal hydraulic model investigation was initiated by the Coastal Engineering Research Center, US Army Engineer Waterways Experiment Station. The purpose of the study was to evaluate the stability and general overtopping conditions for the proposed rehabilitation design for the lower portion of the NB (Sta 36+00 to Sta 52+00) and determine the most economical design that

would provide a stable cross section. A three-dimensional model study of the harbor was conducted to determine the optimum plan to reduce excessive wave activity in the harbor and is reported separately (Bottin and Mize 1990).

PART II: MODEL

Model Design

9. Model tests were conducted at a geometrically undistorted linear scale of 1:35, model to prototype. Scale was selected based on the absolute size of the model breakwater necessary to ensure the preclusion of stability scale effects (Hudson 1975) and the capabilities of the available wave generator to produce required wave heights at modeled water depths. Time relations were scaled according to Froude Model Law (Stevens et al. 1942). The following model-prototype relations were defined in terms of length L and time T :

Characteristic	Dimension	Scale Relations Model:Prototype
Length	L	$L_r = 1:35$
Area	L^2	$A_r = 1:1,225$
Volume	L^3	$V_r = 1:42,875$
Time	T	$T_r = 1:5.92$

where the subscript r denotes the ratio of model to prototype.

10. The specific weight of water used in the model was assumed to be 62.4 pcf, and that of the prototype (seawater) was 64.0 pcf. The specific weights of model breakwater construction materials were assumed to be identical to the prototype, which was reported to be 165 pcf. The model and prototype variables were related by the transference equation of Hudson (1975):

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p} \left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_a)_p - 1}{(S_a)_m - 1}\right]^3 \qquad (1)$$

where

W_a* = weight of an individual armor unit, lb

m, p = model and prototype quantities, respectively

γ_a = specific weight of an individual armor unit, pcf

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* For convenience, symbols and abbreviations are listed in the Notation (Appendix B).

L_m/L_p = linear scale of the model

S_a = specific gravity of an individual armor unit relative to the water in which it is placed, $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of water, pcf

11. Weight of prototype armor stone and their model equivalents used in this study were as follows:

Armor Stone	Prototype wt, tons	Model wt, lb
"A"	11-19	0.442 - 0.764
"A-1"	6-11	0.241 - 0.442
"A-2"	3-8	0.121 - 0.322
Existing Seaside	1-13	0.04 - 0.523
"T"	1-3	0.04 - 0.121
"T-1"	0.1-1	0.004 - 0.04

Test Facilities and Equipment

12. Tests were conducted in a 300-ft-long, 6-ft-wide, 6-ft-deep wave tank. Figure 3 describes the tank dimensions and bottom slopes. The toe of the breakwater section was placed 242 ft from the wave board. Local bathymetry was represented by a 1V on 50H slope for a simulated prototype distance of 1,050 ft (30-ft model) seaward of the breakwater section.

13. Waves were generated by an electronically controlled hydraulic system, which included a piston-type wave board. Displacement of the wave board was controlled by a command signal transmitted to the wave board by a Digital Equipment Corporation (DEC) MicroVax I computer. Waves were produced by the periodic displacement of the wave board. The command signals to drive the wave board were generated on a DEC VAX 3600 computer.

14. Wave data were collected by double-wire resistance-type gages, sampled at 10 Hz. Nine wave gages were used, arranged in three arrays of three gages each, which permitted calculation of incident and reflected wave heights by the method of Goda and Suzuki (1976). Array 1 measured wave heights near the wave board, Array 2 measured wave heights directly seaward (approximately 350 prototype feet) of the structure, and Array 3 measured wave heights directly shoreward of the structure. Wave heights measured at Arrays 2 and 3 were used to compute transmission coefficients. Water surface elevations recorded from the gages were stored on magnetic disk and analyzed using

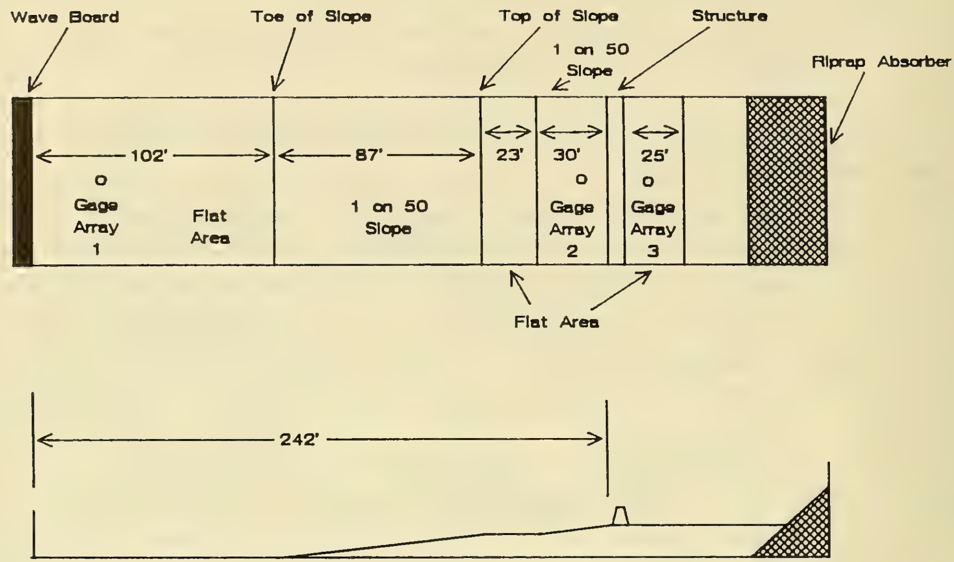

Figure 3. Wave tank used in study

the Time Series Analysis computer program,* which can execute several analysis operations. The operations used in this study were mean downcrossing analysis to obtain significant, maximum, and average wave heights, mean water level, and significant and average wave period at each gage; single channel frequency domain analysis to acquire peak period T_p , zero-moment wave height Hmo , and spectral density plots for each gage; and unidirectional spectral density incident/reflection analysis to determine the incident and reflected parameters at each array.

Test Procedures

15. Design waves were provided by SPL and were based on the annual and extreme wave climate predicted for the project location. Fifteen wave and still-water level (swl) conditions were selected for testing and are listed in Table 1.

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* C. E. Long and D. L. Ward, 1987, "Time Series Analysis," unpublished computer program, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Table 1

Design Wave Conditions, Redondo Beach King Harbor

H_{mo} ft	T_p sec	Still-Water Level ft
13	12	+7
13	14	+7
13	16	+7
13	18	+7
13	16	0*
16	12	+7
16	14	+7
16	16	+7
16	18	+7
16	16	0*
20	14	+7
20	16	+7
20	16	0*
24	14	+7
24	16	+7

* Tests conducted Plan 3 only.

16. Command signals were generated to simulate the Texel Marsen Arsloe shallow-water spectrum (Hughes 1984) for the four design wave periods. The range of design wave heights was obtained by varying the percentage of the command signal sent to the wave generator. The duration for each test condition was 30 min model time (≈3 hr prototype).

17. The sequence of producing the wave conditions was from shortest to longest wave period beginning with the lowest wave height. Damage was determined after all wave conditions at each set of constant wave height (13-, 16-, 20-, 24-ft) were generated.

18. Prior to installation of the breakwater section, the wave facility was calibrated for the selected wave conditions. Arrays 1 and 2 were located as shown in Figure 3, and Array 3 was positioned so the middle gage was directly above the top of the 1:50 slope. Incident wave measurements from Array 3 gave wave heights at the toe of the breakwater that were not affected by reflections from the breakwater.

Method of Constructing Test Sections

19. Construction of the modeled section simulated prototype construction as closely as possible. The base, core, and secondary armor layers were each placed by dumping from a shovel or bucket to the predetermined grade level. Hand trowels were used to smooth and compact the core material to simulate natural consolidation which would result from wave action during construction of the prototype breakwater. The primary armor layer was placed by hand in a random manner below mllw. Random placement consists of selecting a stone at random and placing it in contact with adjacent stones on the structure, with no attempt to orient the axes of the stone or key the stone to the structure. The placed stone method was used for armor units above mllw. A small group of stones were randomly selected, and placement was made by choosing the stone that would best fit the next position in the armor layer. No attempt was made to key the stone into the structure to any greater extent than would be feasible during prototype construction.

Method of Determining Damage

20. The number method was used to determine damage to a test section. The number of armor units displaced from a test section was counted and expressed as a percentage of the total number of armor units placed in the section before testing. The section was considered "not damaged" if the displaced stone count was less than 2 percent of the total number of units in the section.

21. The stability of the test sections could be calculated from the test results by

$$K_d = \frac{\gamma_a H^3}{W_a (S_a - 1)^3 \cot \theta} \qquad (2)$$

where

K_d = stability coefficient

γ_a = unit weight of the armor units

H (in this study H_{mo}) = highest wave height at the structure that causes no damage; i.e., wave height at which damage $\leq$ 2 percent

W_a = weight of an individual armor unit in the primary cover layer

S_a = specific gravity of armor unit, relative to the water at the structure

θ = angle of the structure slope measure from horizontal in degrees

PART III: TEST RESULTS

Introduction

22. Four plans (Plans 1, 2, 3, and 5) were tested to check the stability of the proposed modifications to the NB from Sta 36+00 to 52+00. Each of these plans was subjected to the series of 12 design wave conditions at +7 ft mllw, and the series was repeated at least once. The structure was rebuilt after each series of 12 wave conditions. The purpose of repeating the tests was to ensure consistency in building the breakwater and to verify results. Displaced stones were counted for the armor units on the sea side of the structure and for armor units on the rehabilitation sections. Three wave conditions were generated with Plan 4 installed to check transmitted wave heights. This part of the report describes the plans tested and results from each plan.

Plan 1

Description

23. Plan 1 (Figure 4, Photos 1-6), proposed by SPL, consisted of raising the crest elevation to +20 ft mllw by adding one layer of 11- to 19-ton stone ("A" Stone) to the existing crest, adding two layers of "A" Stone to the harbor side slope from -10 ft mllw to the crest, and placing 3- to 8-ton stone ("A-2" Stone) to -10 ft mllw from the base of the breakwater on the harbor side to form a toe buttress. Placement of armor units on the harbor side at King Harbor is more economical because shallower water depths result in less armor stone required and armor stone placement by ocean-borne construction equipment is protected by the breakwater.

Results

24. Plan 1 was subjected to the 12 wave conditions at +7.0 ft mllw and repeated twice. Figures 5 and 6 illustrate percent damage for sea and harbor sides of the breakwater, respectively, as a function of the incident wave height $(H_{mo})_i$. Damage to the sea side increased dramatically when $(H_{mo})_i > 20$ ft , but was 10 percent or less for lower incident waves. Damage

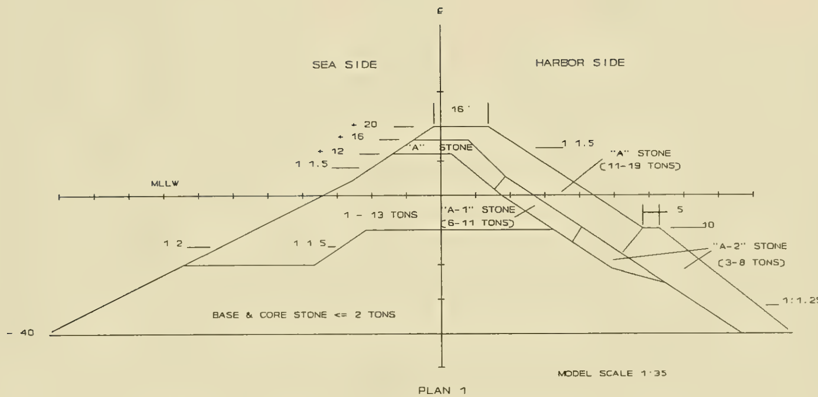

Figure 4. Plan 1, King Harbor stability study

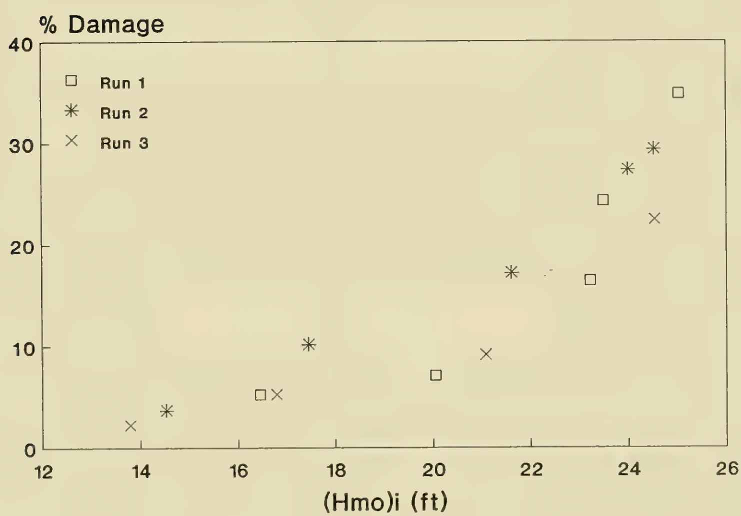

Figure 5. Sea-side damage, Plan 1

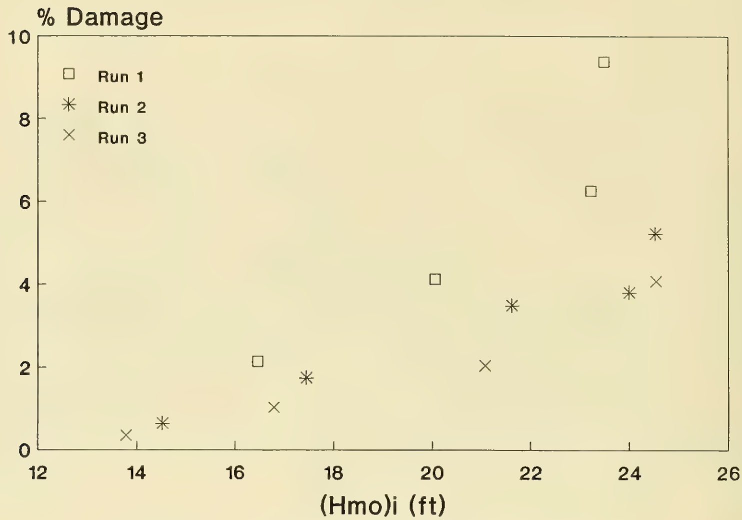

Figure 6. Harbor-side damage, Plan 1

to the proposed rehabilitation section (harbor side) was less than 2 percent and was considered "not damaged" for $(H_{mo})_i < 17$ ft .

25. Transmitted wave heights $(H_{mo})_t$ resulted from waves overtopping the structure and wave transmission through the structure. Transmission data for Plan 1 are listed in Table 2. Transmission coefficients $K_t = (H_{mo})_i/(H_{mo})_t$ ranged from 0.17 to 0.32. Transmitted wave height plotted as a function of incident wave height for Plan 1 is shown in Figure 7. (A nondimensional plot of wave transmission is located in Appendix A, Figure A1.) Wave heights on the harbor side approached 8 ft for the highest incident waves.

Summary

26. The tests indicated that the proposed section was stable for design wave conditions. The sea-side section suffered severe damage with the highest waves, but damage ≤10 percent for $(H_{mo})_i \leq 20$ ft . Results from Plan 1 indicated the structure might be stable with smaller armor stones placed on the middle section of the harbor side.

Table 2

Plan 1 Wave Heights

T sec	$(H_{mo})_i$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
12	13.5	12.0	2.1	0.17
12	14.8	13.5	2.7	0.20
12	16.1	14.5	2.5	0.18
12	16.9	15.3	2.6	0.17
12	18.4	16.7	3.5	0.21
12	19.7	17.3	3.4	0.20
14	14.4	13.7	2.7	0.20
14	14.6	13.2	2.8	0.21
14	15.7	14.4	3.0	0.21
14	17.5	15.9	3.4	0.21
14	17.6	16.6	3.5	0.21
14	19.3	17.3	3.8	0.22
14	21.6	19.4	4.5	0.23
14	22.8	21.2	5.4	0.25
14	25.0	22.2	5.7	0.26
14	25.8	22.6	5.7	0.25
14	27.0	23.6	6.1	0.26
14	27.4	24.2	6.3	0.26
16	14.2	14.1	3.1	0.22
16	14.7	14.1	3.1	0.22
16	15.0	14.5	3.0	0.21
16	17.0	16.6	3.8	0.23
16	17.4	16.6	3.7	0.22
16	18.3	17.6	4.0	0.23
16	22.0	21.0	5.8	0.28
16	22.0	20.7	5.2	0.25
16	22.7	21.0	5.7	0.27
16	25.0	23.4	7.0	0.30
16	25.0	23.3	7.0	0.30
16	25.4	23.6	7.0	0.30
16	25.5	23.7	7.0	0.30
16	25.8	23.8	6.3	0.26
16	26.8	24.4	7.2	0.30
16	26.9	24.9	7.4	0.30
16	27.0	24.5	7.8	0.32
16	27.6	25.0	7.5	0.30
16	28.1	25.1	7.8	0.31
18	14.2	13.9	3.3	0.24
18	14.5	14.1	3.0	0.21
18	15.7	15.2	3.7	0.24
18	17.4	17.1	4.2	0.25
18	17.5	16.9	4.1	0.24
18	19.2	18.5	4.8	0.26

* Incident wave height at the wave board.

** Incident wave height at the structure.

† Transmitted wave height approximately 350 ft behind the structure.

†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

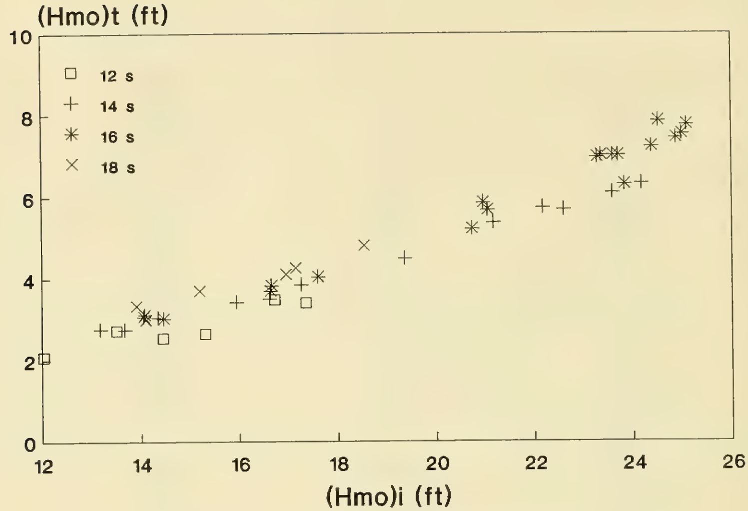

Figure 7. Wave transmission, Plan 1

Plan 2

Description

27. Plan 2 consisted of the same geometric cross section as Plan 1, but 6- to 11-ton armor stone ("A-1" Stone) was placed from +10 ft mllw to -10 ft mllw, with "A" Stone placed from +10 ft mllw to the crown on the harbor side (Figure 8, Photos 7-12). "A-1" Stone was chosen by SPL as the most economical armor size to replace the larger armor units. The toe buttress was composed of "A-2" Stone, identical to Plan 1.

Results

28. The design wave conditions were tested for Plan 2, and repeated once. All tests were conducted at +7.0 ft mllw. Percent damage was determined for the sea-side, "A" Stone, and "A-1" Stone sections. Figure 9 shows sea-side damage similar to Plan 1. Damage for $(H_{mo})_i \leq 20$ ft was approximately 10 percent, but damage increased to 20 to 25 percent for $(H_{mo})_i \geq 24$ ft . Figure 10 shows that the crown suffers little damage and was intact for $(H_{mo})_i < 22$ ft ; however, damage increased to 30 to 40 percent for $(H_{mo})_i \geq 24$ ft . The displacement of armor units from this section was caused by the heavy overtopping associated with the highest waves. Damage to the

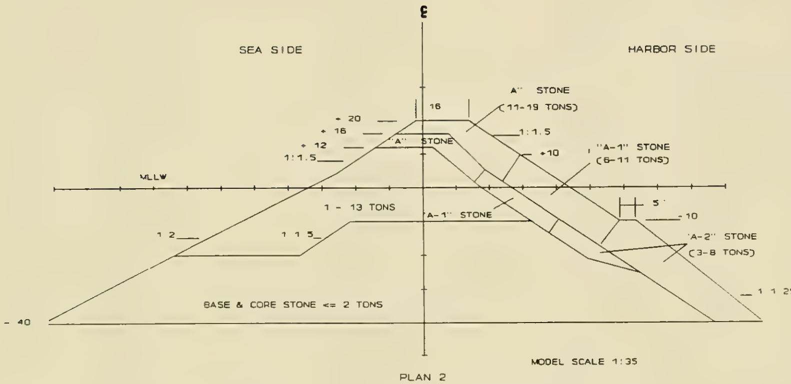

Figure 8. Plan 2, King Harbor stability study

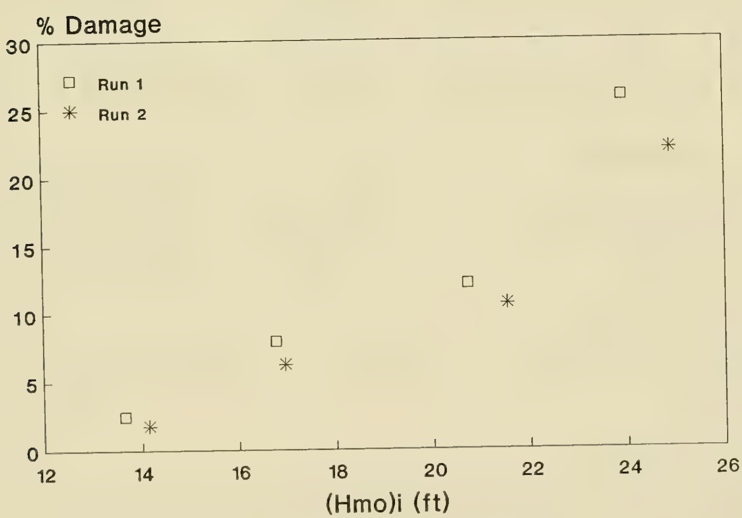

Figure 9. Sea-side damage, Plan 2

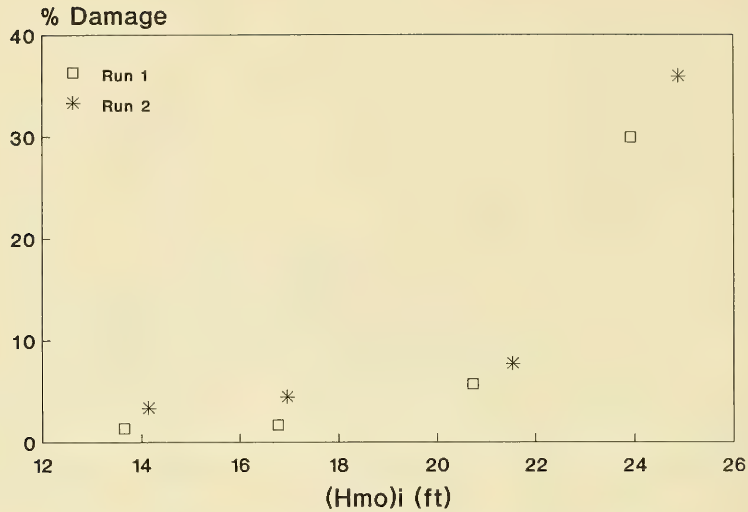

Figure 10. Harbor-side damage, Plan 2, "A" Stone

middle section ("A-1" Stone section) reached 5.1 percent for a 24-ft wave, but was not damaged for $(H_{mo})_i \leq 22$ ft (Figure 11).

29. Transmitted wave heights with Plan 2 installed were similar to Plan 1. Transmission coefficients for Plan 2, listed in Table 3, ranged from

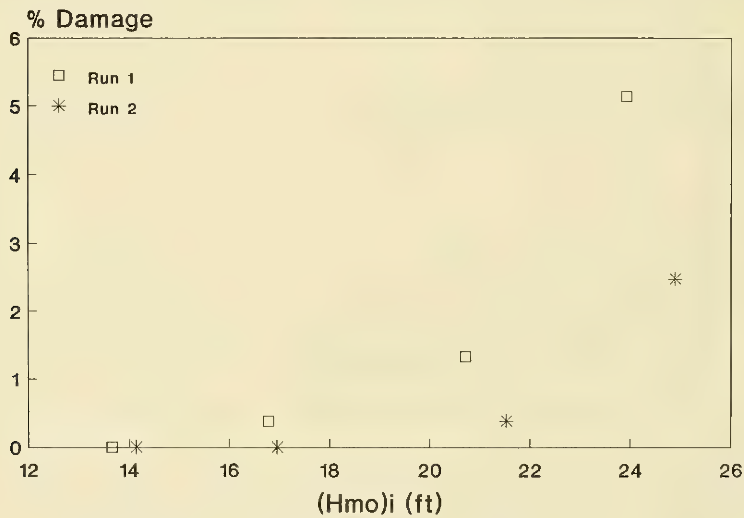

Figure 11. Harbor-side damage, Plan 2, "A-1" Stone

Table 3

Plan 2 Wave Heights

T sec	$(H_{mo})_1$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
12	14.9	13.5	2.5	0.18
12	14.9	13.5	2.8	0.21
12	18.8	16.9	3.6	0.21
12	19.1	17.3	3.3	0.19
14	14.2	13.3	2.6	0.20
14	14.6	13.9	3.2	0.23
14	17.9	16.7	4.0	0.24
14	18.1	17.0	4.1	0.24
14	22.6	20.6	5.7	0.27
14	23.1	21.6	5.9	0.27
14	27.1	23.6	7.4	0.31
14	28.0	24.6	7.7	0.31
16	14.1	13.7	3.0	0.22
16	14.8	14.6	3.4	0.24
16	16.9	16.3	4.0	0.25
16	17.6	17.2	4.4	0.25
16	22.0	20.8	6.1	0.29
16	22.4	21.5	6.4	0.30
16	26.5	24.3	7.9	0.33
16	27.3	25.1	8.4	0.33
18	14.5	14.1	3.4	0.24
18	14.7	14.5	3.6	0.25
18	17.3	16.7	4.5	0.27
18	17.4	16.9	4.5	0.27

* Incident wave height at the wave board.
** Incident wave height at the structure.
† Transmitted wave height approximately 350 ft behind the structure.
†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

0.18 to 0.33. Wave heights on the harbor side of the breakwater were as high as 8.45 ft, Figure 12 (see Appendix A, Figure A2 for nondimensional plot).

Summary

30. Results from Plan 2 indicated the "A-1" Stone was stable placed at the middle harbor-side section. Armor stone in the upper section was stable for 22 ft waves, but the section was severely damaged when $(H_{mo})_i \geq 24$ ft . Damage to the sea side was 10 to 15 percent for $(H_{mo})_i < 22$ ft and was as high as 25.8 percent for 24-ft waves.

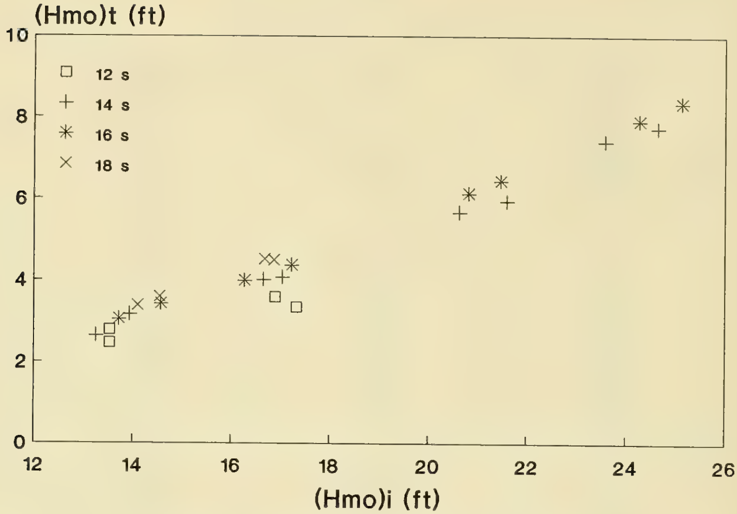

Figure 12. Wave transmission, Plan 2

Plan 3

Description

31. Results from Plan 2 indicated that the "A-1" Stone was stable for all waves if placed on the middle section of the harbor side; "A" Stone in the upper section was stable for $(H_{mo})_i < 22$ ft ; and stability tests with smaller armor stone placed in the upper section might be stable. Plan 3 (Figure 13, Photos 13-18) consisted of the same geometry as Plan 1, except "A-1" Stone was placed on the harbor-side slope from the breakwater crest to -10 ft mllw. The harbor-side toe buttress remained the same as previous plans.

Results

32. Plan 3 was subjected to the 12 design waves at +7.0 ft mllw and repeated. During the repeat test, the regular series of test waves for a constant wave height was conducted at a swl of +7 ft mllw, and the damage was assessed; then the water level was dropped to 0.0 ft mllw and that constant wave height repeated for a 16-sec wave period (see Table 1) and damage assessed again. Wave action at the low water contributed to the overall damage on the sea side (Figure 14) but did not cause significant instability of

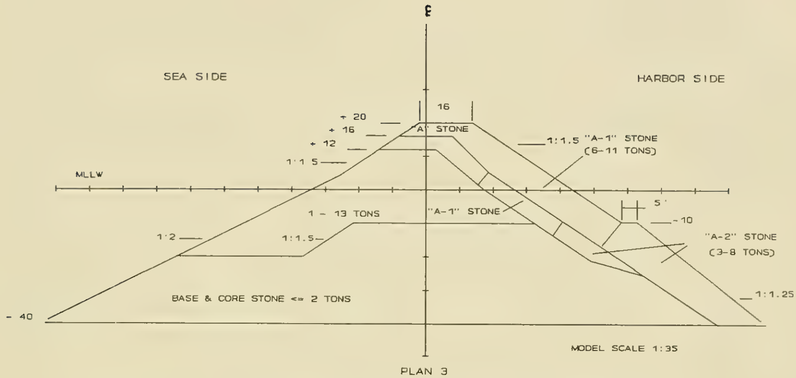

Figure 13. Plan 3, King Harbor stability study

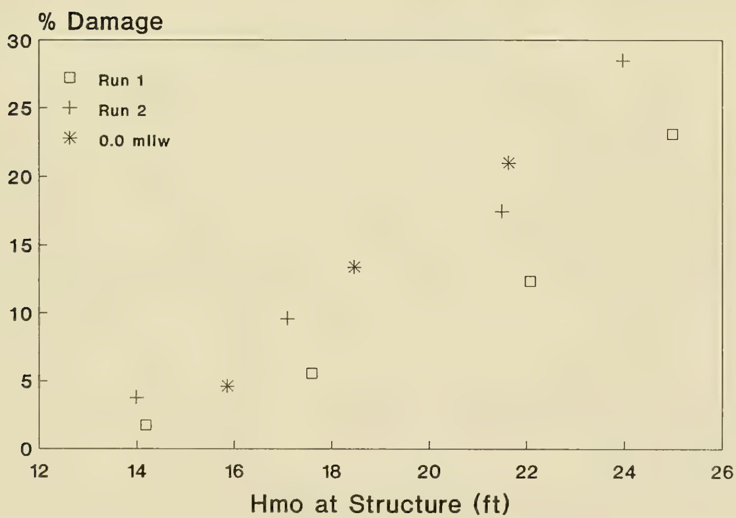

Figure 14. Sea-side damage, Plan 3

the toe stone. Damage to the harbor side, including damage caused by waves at low water, is shown in Figure 15. Waves generated at low water caused little damage to the harbor side. The harbor-side section was not damaged for $(H_{mo})_i \leq 18$ ft , and maximum damage was approximately 6 percent.

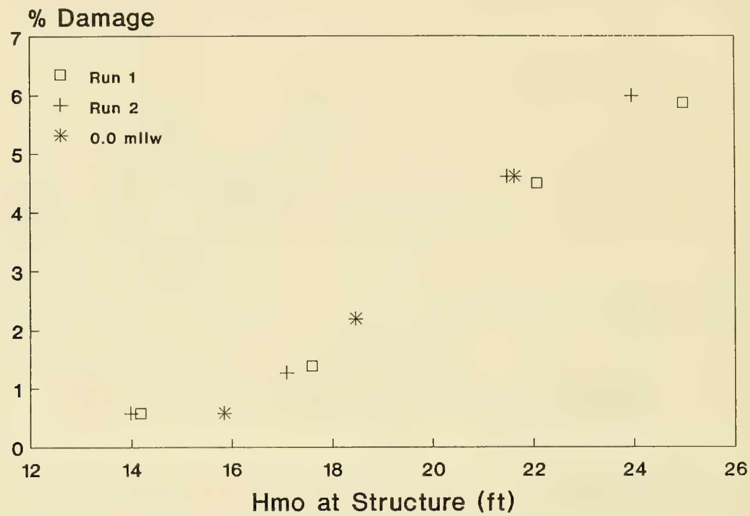

Figure 15. Harbor-side damage, Plan 3

33. Plan 3 consisted of only one rehabilitation section covering the harbor side; therefore, some stones near the crest which roll onto a lower part of the rehabilitation section are not counted as displaced stones. Because Plan 2 gave an indication of the crest stability, it was desirable to separate Plan 3 into sections to determine the performance of smaller armor units placed at the crest. To compare Plan 3 with Plan 2, the harbor-side section was divided into two zones, and the number of armor units displaced from each zone was recorded during the repeat test of Plan 3. Zones 1 and 2 were defined as the rehabilitation sections from the seaward end of the crown to +10 ft mllw, and from +10 to -10 ft mllw, respectively. Figure 16 shows percent damage to Zone 1 versus $(H_{mo})_i$. Damage to Zone 1 was 10 percent or less for $(H_{mo})_i < 22$ ft , but for the 24-ft wave, damage was 63 percent. The increase in damage is a result of heavy overtopping of the structure. Damage to Zone 2 was minor for the 24-ft wave and not damaged for lower waves (Figure 17).

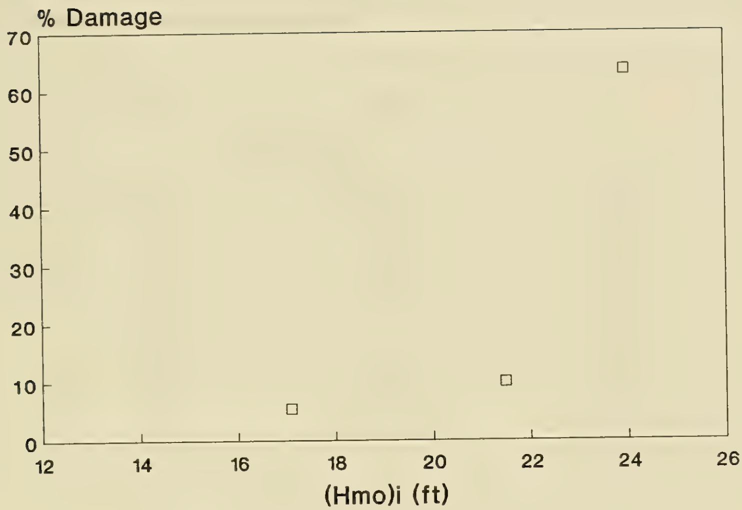

Figure 16. Zone 1 damage, Plan 3

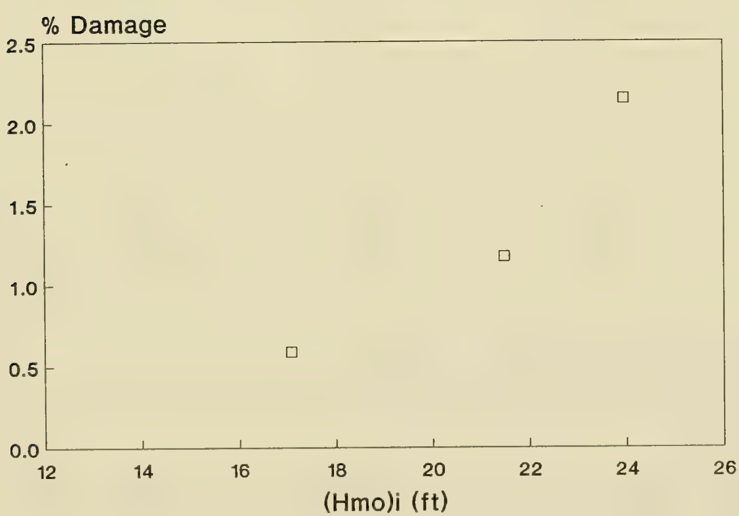

Figure 17. Zone 2 damage, Plan 3

34. Transmitted wave heights and K_t-values for Plan 3 at +7.0 ft mllw are listed in Table 4. Harbor-side wave heights reached 8.1 ft for the highest incident waves (Figure 18). Wave transmission with Plan 3 installed was comparable to Plans 1 and 2. A plot of nondimensional wave transmission for Plan 3 is located in Appendix A, Figure A3.

Table 4

Plan 3 Wave Heights

T sec	$(H_{mo})_1$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
12	14.9	13.7	2.6	0.19
12	15.3	14.2	2.5	0.18
12	18.7	17.4	3.3	0.19
12	19.0	17.2	3.0	0.18
14	14.3	13.6	3.1	0.22
14	15.3	14.5	3.1	0.21
14	17.8	16.8	3.8	0.23
14	18.8	18.0	4.1	0.23
14	23.0	21.3	5.5	0.26
14	24.3	22.4	5.8	0.26
14	27.3	24.7	6.9	0.28
14	27.3	23.8	7.6	0.32
16	14.4	13.8	3.1	0.22
16	14.9	14.2	3.0	0.21
16	17.8	16.9	3.9	0.23
16	18.2	17.2	4.1	0.24
16	23.1	21.6	5.6	0.26
16	23.5	21.7	5.8	0.27
16	26.4	24.1	8.1	0.34
16	27.9	25.2	8.0	0.32
18	14.7	14.2	3.4	0.24
18	14.9	14.4	3.1	0.21
18	18.0	17.5	4.3	0.25
18	18.5	17.8	4.6	0.26

* Incident wave height at the wave board.
** Incident wave height at the structure.
† Transmitted wave height approximately 350 ft behind the structure.
†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

35. Wave transmission was considerably less for waves generated at 0.0 ft mllw (Table 5, Figure 19). Wave overtopping was less because of higher freeboard (the distance from the breakwater crest to the swl). The breakwater was also wider at +0.0 ft mllw, which resulted in more energy dissipation and less transmission through the breakwater.

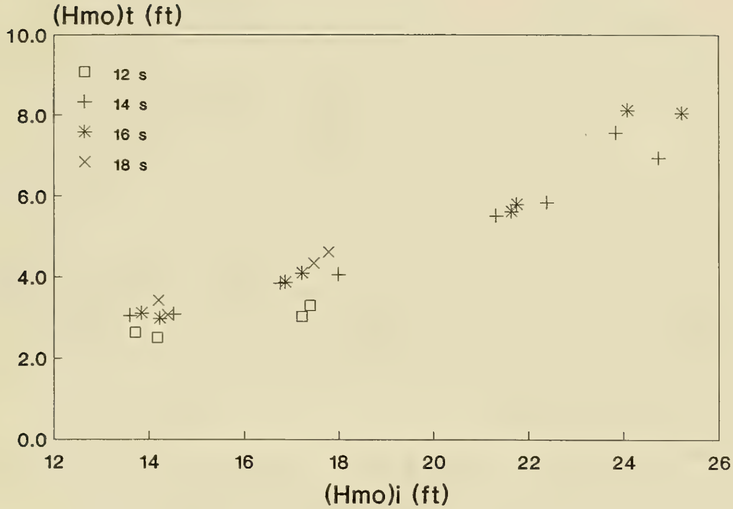

Figure 18. Wave transmission, Plan 3

Table 5

Plan 3 Wave Heights, +0.0 mllw

T sec	$(H_{mo})_1$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
16	16.6	15.3	2.2	0.14
16	19.8	17.9	2.5	0.14
16	24.2	21.0	3.6	0.17

* Incident wave height at the wave board.
** Incident wave height at the structure.
† Transmitted wave height approximately 350 ft behind the structure.
†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

Summary

36. The rehabilitation section was stable for the wave conditions; however, the crest section suffered 63-percent damage for $(H_{mo})_i \geq 22$ ft. The sea side had damage similar to Plans 1 and 2. Results of the low-water wave tests showed that the toe area was stable for the wave conditions generated. Wave transmission with Plan 3 was also similar to transmission with Plans 1 and 2; however, the transmitted waves for Plans 1-3 were higher than desired by SPL.

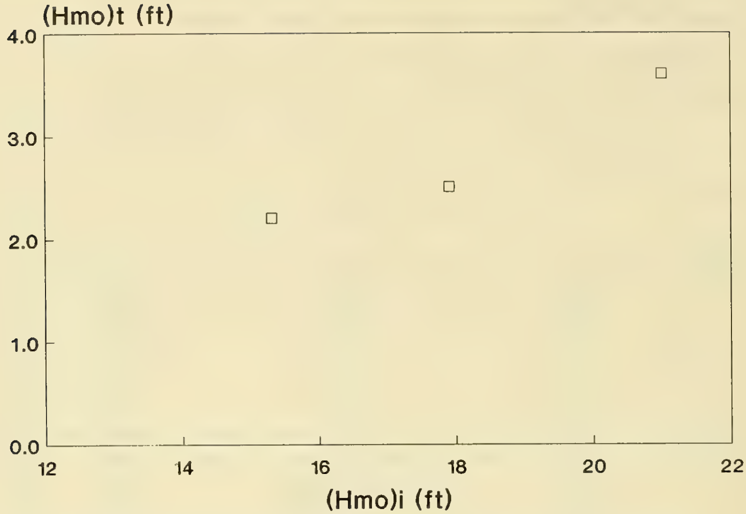

Figure 19. Wave transmission, Plan 3, +0.9 ft mllw

Transmission Test

37. It was uncertain whether the excessive wave heights on the harbor side were mostly by transmission through the structure or by wave overtopping. In order to separate transmitted and overtopped energy, a sheet of metal, bent at an upward angle, was mounted on the crown of the Plan 3 structure, and three wave conditions were run. The sheet metal rested flat on the breakwater crown to allow waves to overtop the crown, but the angled section prevented overtopped waves from entering the harbor side. This configuration allowed waves to overtop the structure as they would under normal conditions, but prevented additional hydrostatic pressure to build on the seaward side, which could force energy through the structure. Therefore, only waves transmitted through the breakwater were measured on the harbor side.

38. It is generally felt that the longer wave periods transmit more wave energy; thus, wave heights using the longest wave periods within each of the designated design groups (Table 1) were tested. They were 16-ft, 18-sec; 18-ft, 18-sec; and 20 ft, 16-sec waves at a swl of +7 ft mllw. The results are listed in Table 6, and transmitted wave height is plotted versus incident wave height in Figure 20. The tests indicated that $K_t \approx 0.20$ for waves

Table 6

Plan 3 Wave Heights, Transmission Test

T sec	$(H_{mo})_1$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
18	14.4	15.0	3.0	0.20
18	17.9	18.6	3.4	0.18
16	21.6	20.8	4.0	0.19

* Incident wave height at the wave board.
** Incident wave height at the structure.
† Transmitted wave height approximately 350 ft behind the structure.
†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

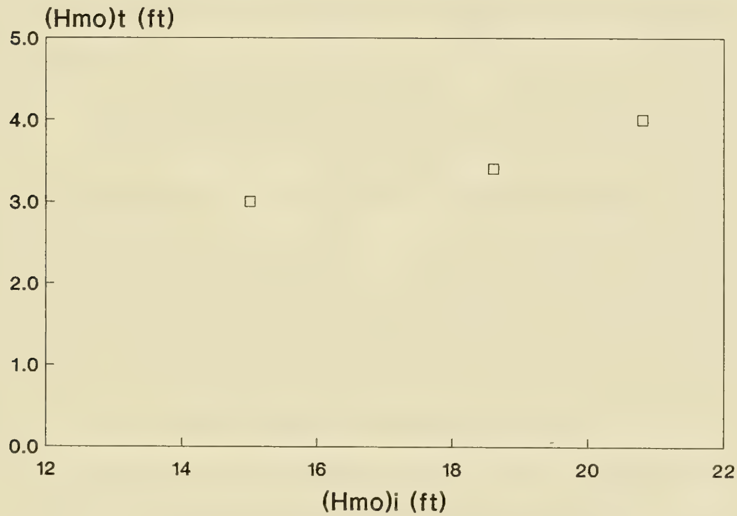

Figure 20. Wave transmission through breakwater, Plan 3

transmitted through the breakwater. The transmission test showed that overtopping contributes to excessive wave heights in the harbor, but most of the wave energy is transmitted through the structure.

Plan 4

39. Plan 4 (Figure 21) consisted of adding a 9-ft-wide layer of 1- to 3-ton stone ("T" Stone) to the existing structure on the harbor side and

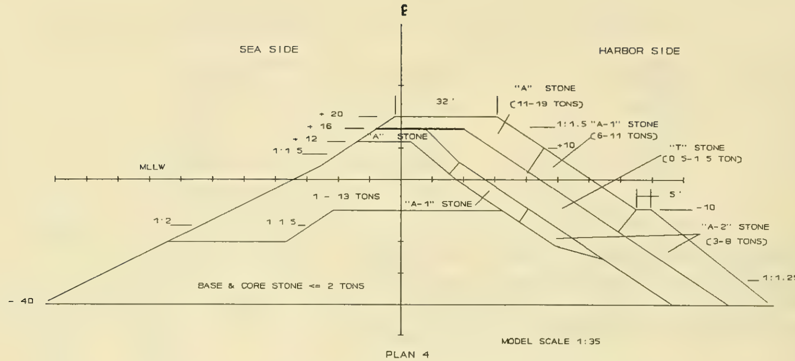

Figure 21. Plan 4, King Harbor stability study

placing "A" Stone from the crown to +10 ft mllw, and "A-1" Stone from +10 to -10 ft mllw. The layer of smaller stone was placed to filter energy transmitted through the breakwater. The crown height remained at +20 ft mllw, and the harbor-side toe buttress was the same as in previous plans.

40. The structure was subjected to three wave heights (13, 16, and 20 ft) for a 16-sec period at +7.0 ft mllw prior to checking stability with the 12 design conditions to determine if the structure lowered transmission to an acceptable level. The transmission coefficients, listed in Table 7, were higher than desired by SPL; therefore, stability tests were not conducted for this plan (see Appendix A, Figure A4 for a nondimensional plot of K_t).

Table 7

Plan 4 Wave Heights, Initial Test

T sec	$(H_{mo})_1$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
16	14.1	13.4	2.3	0.17
16	17.0	15.9	2.8	0.18
16	22.2	20.9	4.3	0.20

* Incident wave height at the wave board.
** Incident wave height at the structure.
† Transmitted wave height approximately 350 ft behind the structure.
†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

Plan 5

Description

41. Plan 5 (Figure 22, Photos 19-33) was similar to Plan 4 but consisted of a 10-ft-thick transition layer of 200- to 2,000-lb stone ("T-1" Stone). The armor layers on the harbor side were the same as in Plan 4.

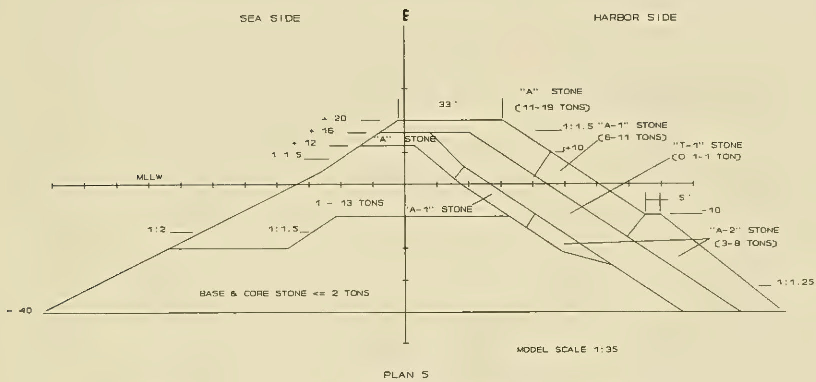

Figure 22. Plan 5, King Harbor stability study

Results

42. Plan 5 was subjected to the three trial wave conditions used with Plan 4, and the resulting transmission coefficients are listed in Table 8. The K_t-values were acceptable; therefore, the structure was repaired and checked for stability using the 12 design wave conditions at +7 ft mllw. The stability tests were repeated once.

Table 8

Plan 5 Wave Heights, Initial Test

T sec	$(H_{mo})_i$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
16	21.9	20.6	3.7	0.18
16	17.3	16.4	2.4	0.15
16	14.5	13.9	2.0	0.14

* Incident wave height at the wave board.
** Incident wave height at the structure.
† Transmitted wave height approximately 350 ft behind the structure.
†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

43. There was concern that the added transition layer would cause added pressure to the sea side of Plan 5 and increase damage. However, Figure 23 shows that damage to the sea side was not significantly higher for Plan 5 than Plans 1-3. Damage to the "A" Stone section of Plan 5 was less than 3 percent for $(H_{mo})_i < 22$ ft and 15 percent for $(H_{mo})_i \approx 24$ ft (Figure 24). The "A-1" Stone section was not damaged for any wave condition during the first run and suffered 5.3-percent damage for the highest wave condition during the repeat test (Figure 25).

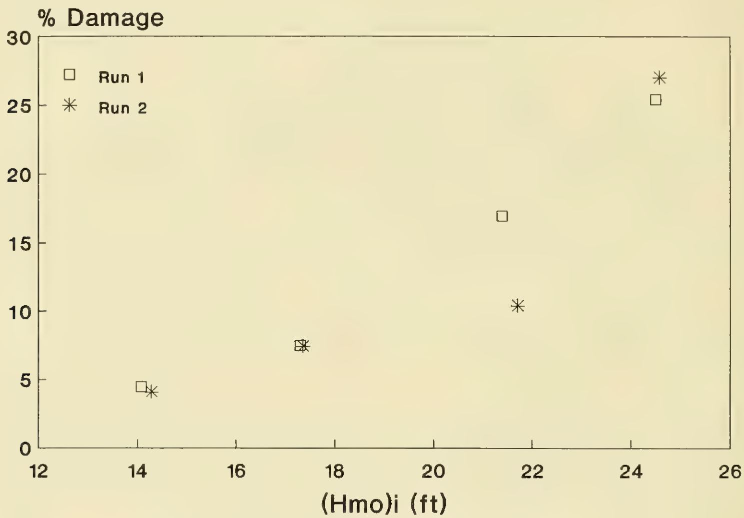

Figure 23. Sea-side damage, Plan 5

44. The maximum transmitted wave height with Plan 5 installed was 5.0 ft (Figure 26; a nondimensional plot of K_t for Plan 5 is located in Appendix A, Figure A5). Transmission coefficients were as high as 0.21 for the highest incident waves, but transmission was less than 18 percent for most wave conditions (Table 9).

<u>Summary</u>

45. The transition layer added to reduce transmission resulted in a wider structure and, therefore, a wider crown. The wider crown improved stability to the rehabilitation sections. Sea-side damage to Plan 5 was similar to Plans 1-3, which indicates the transition layer did not have a considerable effect on damage to the sea side.

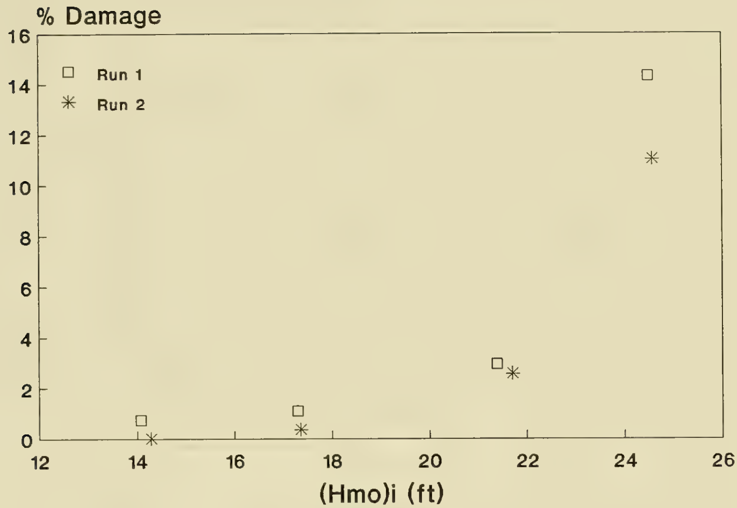

Figure 24. Harbor-side damage, Plan 5, "A" Stone

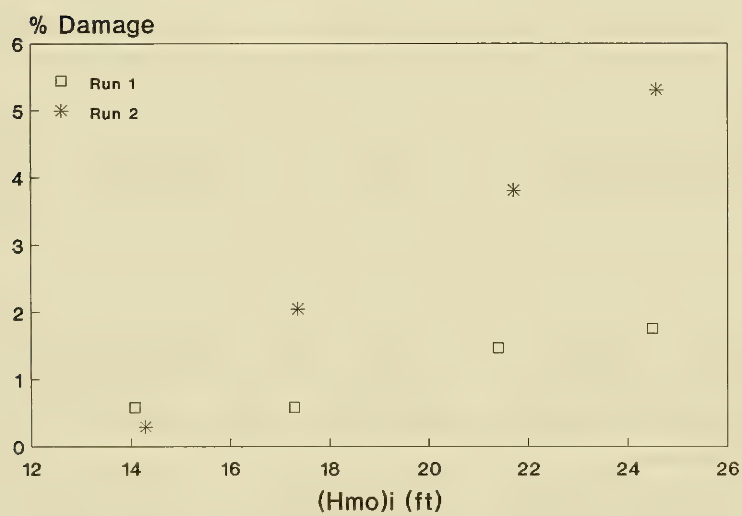

Figure 25. Harbor-side damage, Plan 5, "A-1" Stone

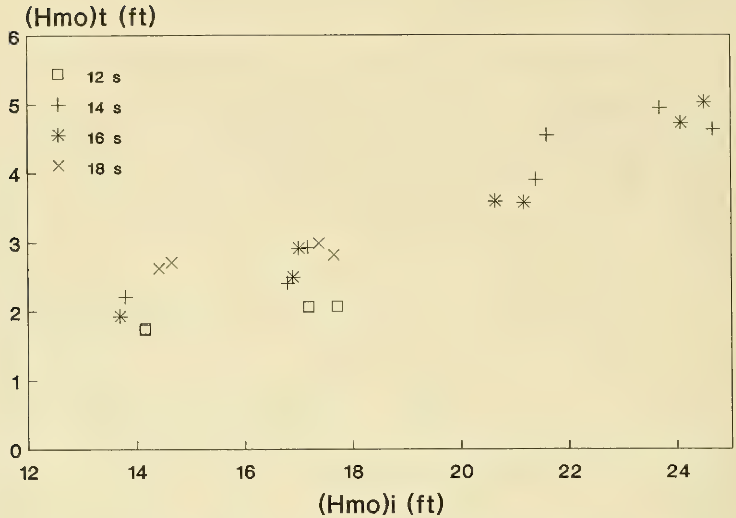

Figure 26. Wave transmission, Plan 5

Overtopping

46. Observations during tests showed that overtopping was essentially the same for Plans 1-3. These plans had the same geometrical shape, and only the stone sizes in the rehabilitation area differed. Overtopping was minor with 13-ft incident waves for all wave periods. Moderate overtopping was observed with 16-ft waves for all periods with occasional solid water going over the structure with 16- and 18-sec periods. Overtopping was heavy with 20- and 24-ft waves for all wave periods.

47. The inclusion of a transition layer in Plan 5 resulted in a wider crown, which reduced the amount of solid water overtopping the structure. Overtopping was minor with 13-ft waves and minor to moderate with 16-ft waves for all periods with Plan 5. Moderate overtopping occurred with 20-ft waves with occasional solid water overtopping the structure. Overtopping was heavy with 24-ft waves.

Table 9

Plan 5 Wave Heights

T sec	$(H_{mo})_i$* ft	$(H_{mo})_i$** ft	$(H_{mo})_t$† ft	K_t††
12	15.1	14.1	1.7	0.12
12	15.8	14.2	1.8	0.12
12	19.1	17.2	2.1	0.12
12	19.1	17.7	2.1	0.12
14	14.4	13.8	2.2	0.16
14	18.4	17.2	2.9	0.17
14	18.1	16.8	2.4	0.14
14	23.4	21.6	4.5	0.21
14	23.5	21.4	3.9	0.18
14	27.7	23.7	4.9	0.21
14	28.2	24.7	4.6	0.19
16	14.2	13.7	1.9	0.14
16	18.0	16.9	2.5	0.15
16	17.8	17.0	2.9	0.17
16	22.9	21.2	3.6	0.17
16	22.7	20.6	3.6	0.18
16	27.3	24.1	4.7	0.20
16	27.6	24.5	5.0	0.20
18	14.9	14.4	2.6	0.18
18	14.8	14.6	2.7	0.18
18	18.1	17.4	3.0	0.17
18	18.3	17.6	2.8	0.16

* Incident wave height at the wave board.
** Incident wave height at the structure.
† Transmitted wave height approximately 350 ft behind the structure.
†† Transmission coefficient, $(H_{mo})_i/(H_{mo})_t$.

Summary of Test Results

48. The tested sections were stable for wave conditions in which $(H_{mo})_i < 22$, but higher waves destroyed the structure for all plans. Percent damage to the rehabilitation section was similar for Plans 1 and 3. These plans consisted of one armor unit size in the rehabilitation section; therefore, displaced stones were counted from only one section. Plan 2 involved two armor sizes on the harbor side. The middle section, consisting of "A-1" Stone, was stable for all wave conditions, but the upper section, consisting of "A" Stone, suffered 30- to 40-percent damage for $(H_{mo})_i > 22$ ft . Plan 3

was divided into middle and upper zones to compare stability of the crest with Plan 2. Percent damage to the upper zone of Plan 3 was comparable to Plan 2, in which $(H_{mo})_i < 22$ ft . However, percent damage increased to 63 percent for $(H_{mo})_i > 22$ ft , which is almost twice the damage with "A" Stone. Plan 5 was identical to Plan 2 in that it consisted of "A" Stone in the upper section and "A-1" Stone in the middle section, except a transition layer of smaller stones was added to reduce wave transmission through the structure. The wider cross section of Plan 5 improved the stability of the rehabilitation sections.

49. Stability coefficients were calculated for each test section (Table 10). The wave height used in Equation 2 was the average of all runs at

Table 10

Stability Coefficients

Plan	Area	$(H_{mo})_i$ ft	W_{50} tons	K_d
1	Sea	13.8	11.0	3.3
1	Harbor	18.2	15.0	5.6
2	Sea	13.7	11.0	3.3
2	A	16.5	15.0	4.2
2	A-1	22.5	8.5	19.3
3	Sea	13.7	11.0	3.3
3	Harbor	18.2	8.5	9.9
3	Z1	13.8	8.5	4.3
3	Z2	23.5	8.5	21.4
4	Sea	-	-	-
4	A	-	-	-
4	A-1	-	-	-
5	Sea	12.0	11.0	2.2
5	A	21.0	15.0	8.6
5	A-1	21.2	8.5	15.7

which 2-percent damage occurred for the test section. Stability coefficients were identical on the sea side for Plans 1-3, but K_d was lower for Plan 5. This may be an effect of additional back pressure caused by the transition layer included in Plan 5. The stability coefficient on the harbor side was higher for Plan 3 than Plan 1, although Plan 3 consisted of lighter armor stone in the section. The highest nondamaging wave was identical for both sections, which means that the Plan 3 section was not necessarily more stable than the Plan 1 section, but the reduction of armor weight from "A" to "A-1"

Stone did not result in an increase of structural damage. All plans had high stability coefficients for the harbor-side sections. Plan 5 had the highest K_d of the upper sections; however, the middle section of Plan 5 gave the lowest K_d-value of the middle sections. Figure 25 shows that damage was slightly higher during the repeat test of Plan 5, which gave a significantly lower nondamaging wave height. If Run 1 is used to obtain the 2-percent damage wave height, $K_d = 26.0$, which is comparable to the middle sections of Plans 2 and 3.

50. Transmission coefficients were as high as 0.33 for Plan 1 and 0.34 for Plans 2 and 3. Plan 4 consisted of the same armor stone and geometry of Plan 2, but a 9-ft-thick layer of "T" Stone was placed between the existing structure and the proposed armor stone on the harbor side. Transmitted wave heights were reduced, but not to an acceptable level. Plan 5 was identical to Plan 4, except the transition layer was 10 ft thick and composed of "T-1" Stone. The transmission coefficients for Plan 5 were acceptable.

PART IV: CONCLUSIONS

51. Based on the results of the test conditions reported herein, it was concluded that:

a. Each of the breakwater plans demonstrated varying degrees of acceptable stability for the sea-side and rehabilitation sections depending on the chosen design condition.

b. Tests at low water (0.0 ft mllw) indicate that the sea-side toe area was stable for the conditions tested.

c. Plan 5 yielded the best combination considering stability, wave overtopping, and wave transmission energy. Any back pressure resulting from the filter layer included in Plan 5 did not cause significant damage to the sea side.

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Photo 1. Plan 1, before testing, side view

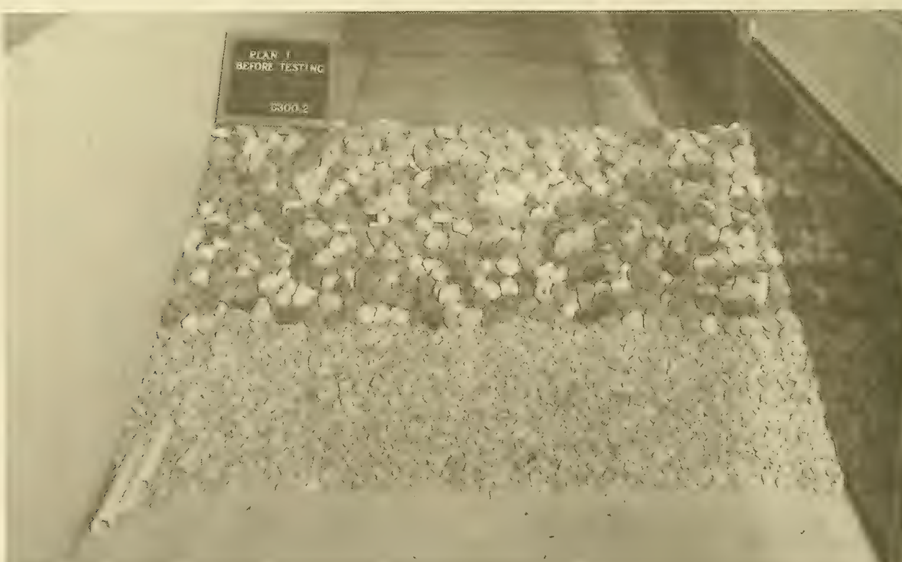

Photo 2. Plan 1, before testing, sea-side view

Photo 3. Plan 1, before testing, harbor-side view

Photo 4. Plan 1, after testing, side view

Photo 5. Plan 1, after testing, sea-side view

Photo 6. Plan 1, after testing, harbor-side view

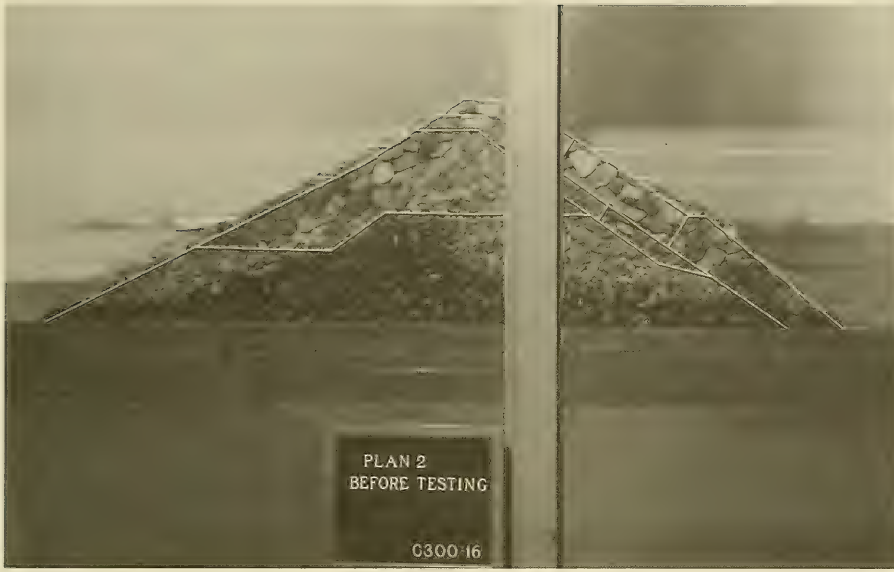

Photo 7. Plan 2, before testing, side view

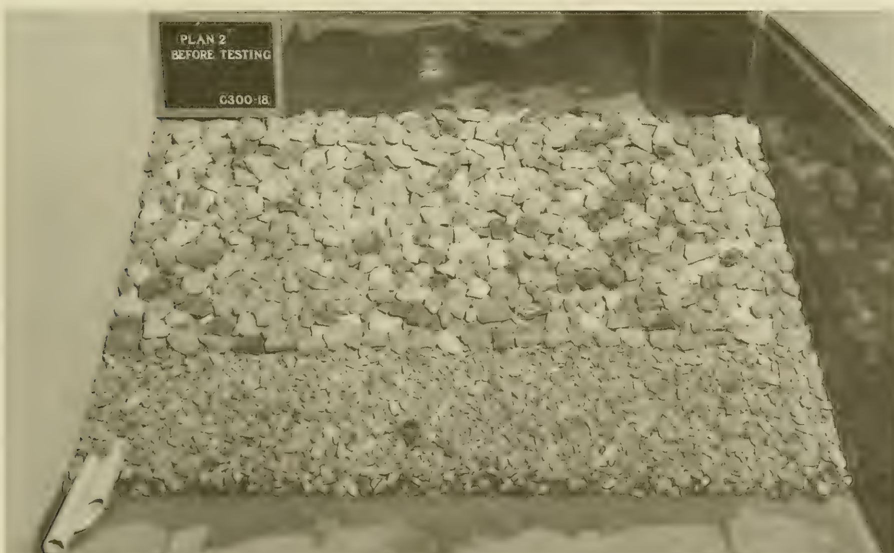

Photo 8. Plan 2, before testing, sea-side view

Photo 9. Plan 2, before testing, harbor-side view

Photo 10. Plan 2, after testing, side view

Photo 11. Plan 2, after testing, sea-side view

Photo 12. Plan 2, after testing, harbor-side view

Photo 13. Plan 3, before testing, side view

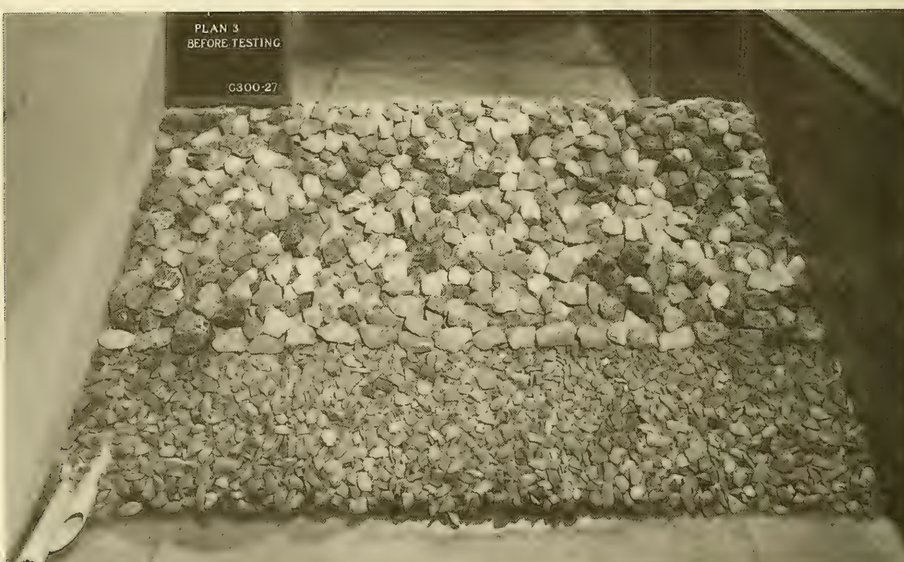

Photo 14. Plan 3, before testing, sea-side view

Photo 15. Plan 3, before testing, harbor-side view

Photo 16. Plan 3, after testing, side view

Photo 17. Plan 3, after testing, sea-side view

Photo 18. Plan 3, after testing, harbor-side view

Photo 19. Plan 5, before testing, side view

Photo 20. Plan 5, before testing, sea-side view

Photo 21. Plan 5, before testing, harbor-side view

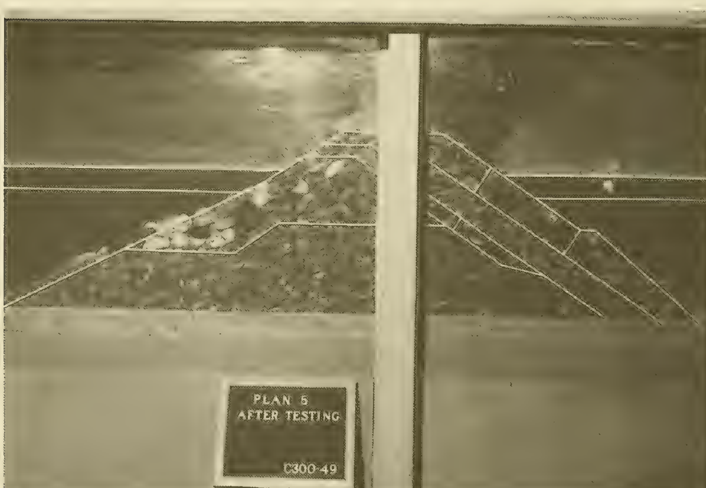

Photo 22. Plan 5, after 13-ft waves, side view

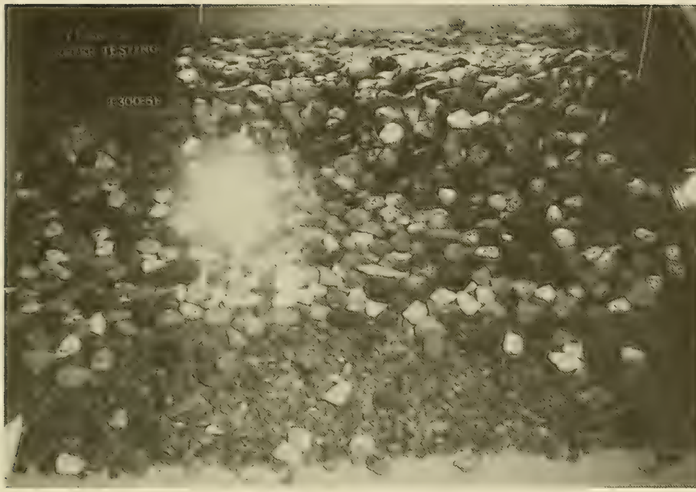

Photo 23. Plan 5, after 13-ft waves, sea-side view

Photo 24. Plan 5, after 13-ft waves, harbor-side view

Photo 25. Plan 5, after 16-ft waves, side view

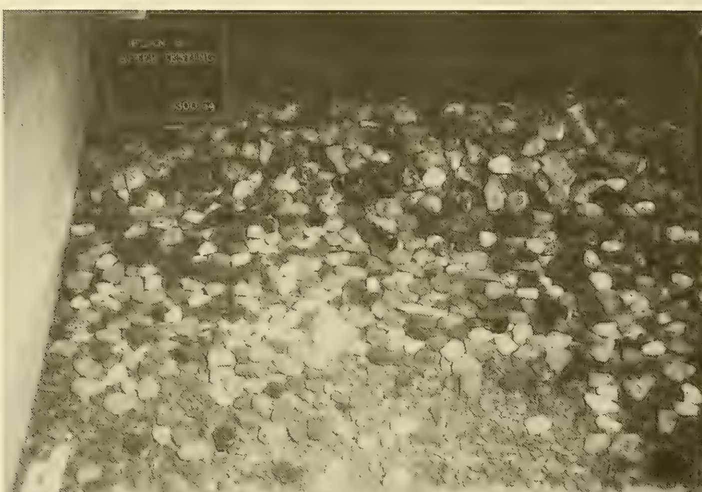

Photo 26. Plan 5, after 16-ft waves, sea-side view

Photo 27. Plan 5, after 16-ft waves, harbor-side view

Photo 28. Plan 5, after 20-ft waves, side view

Photo 29. Plan 5, after 20-ft waves, sea-side view

Photo 30. Plan 5, after 20-ft waves, harbor-side view

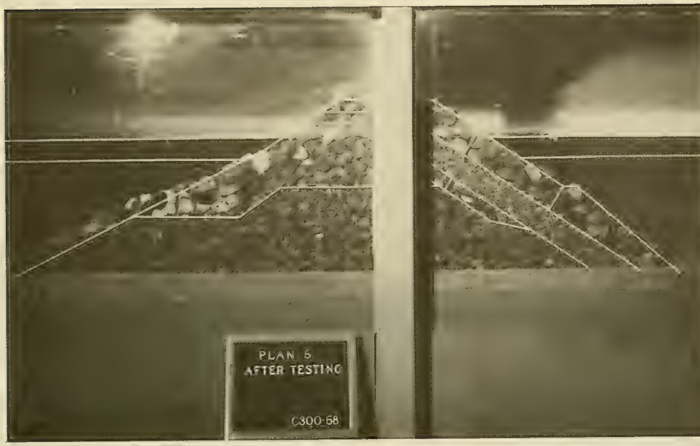

Photo 31. Plan 5, after 24-ft waves, side view

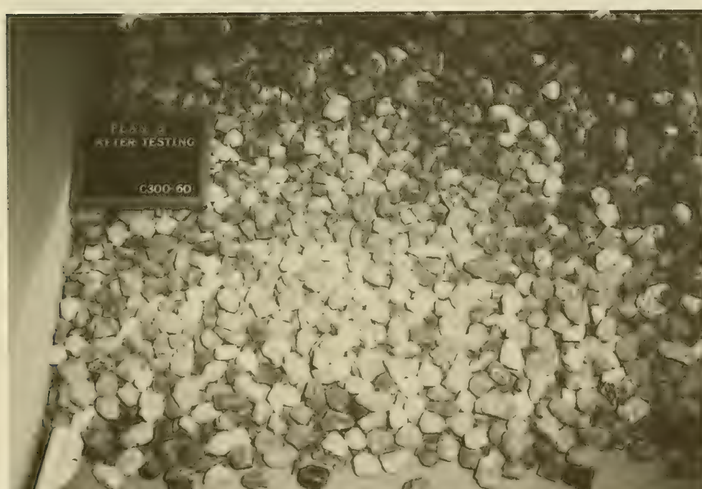

Photo 32. Plan 5, after 24-ft waves, sea-side view

Photo 33. Plan 5, after 24-ft waves, harbor-side view

APPENDIX A: WAVE TRANSMISSION PLOTS

This appendix contains nondimensional plots of the transmission coefficient K_t for each plan tested. Wave transmission is plotted as a function of wave steepness $(H_{mo})_i/(L_p)o$ at the breakwater toe, in which $(L_p)_o$ is the peak deepwater wavelength.

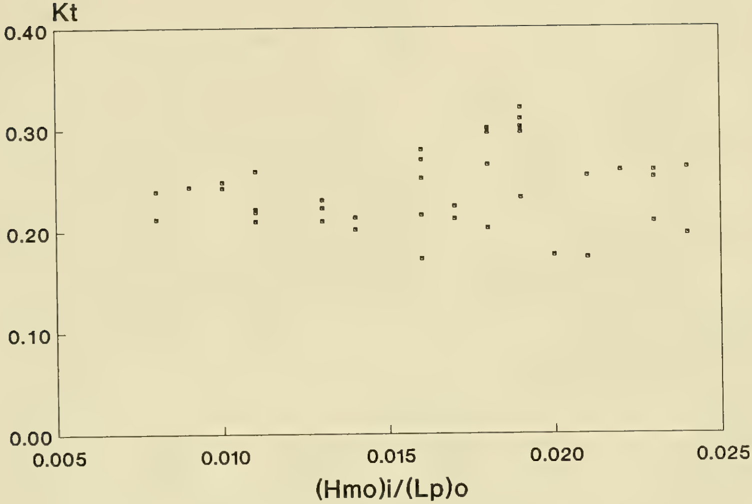

Figure A1. Nondimensional wave transmission, Plan 2

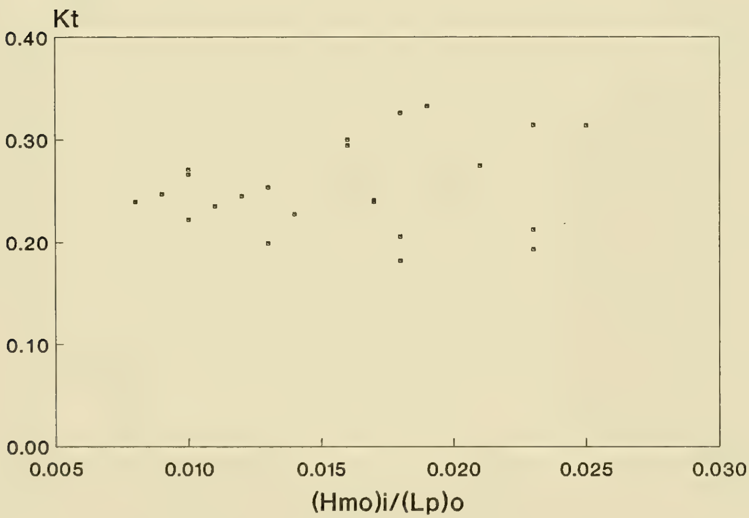

Figure A2. Nondimensional wave transmission, Plan 2

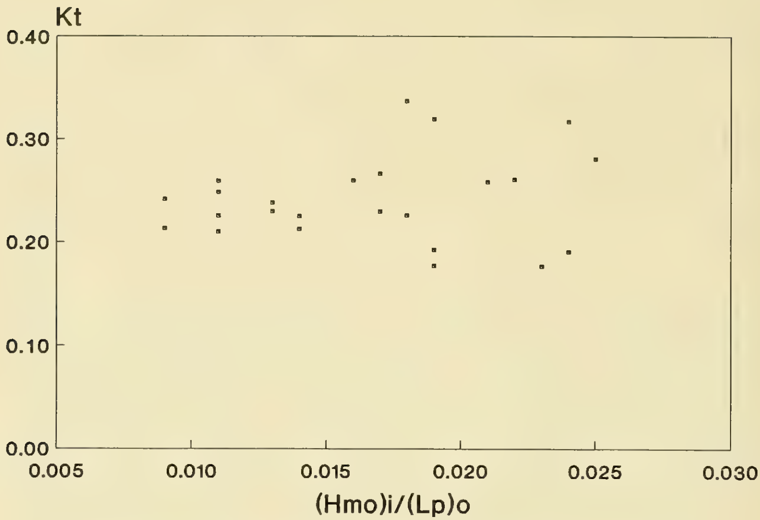

Figure A3. Nondimensional wave transmission, Plan 3

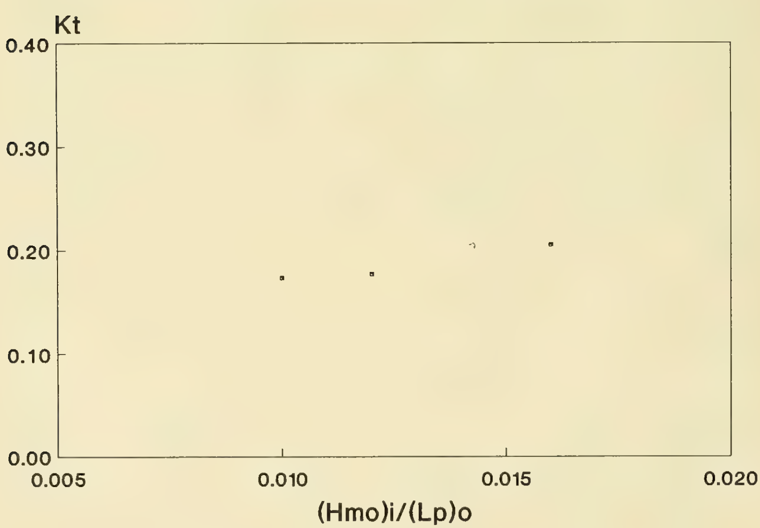

Figure A4. Nondimensional wave transmission, Plan 4

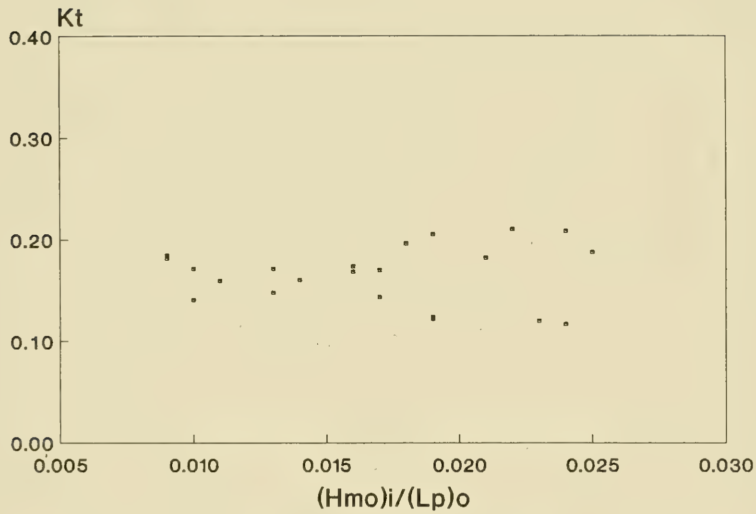

Figure A5. Nondimensional wave transmission, Plan 5

APPENDIX B: NOTATION

H	Highest wave height at structure that causes no damage
H_{mo}	Zero-moment wave height
$(H_{mo})_i$	Incident wave height
$(H_{mo})_t$	Transmitted wave height
K_d	Stability coefficient
K_t	Transmission coefficient
L_m/L_p	Linear scale of the model
$(L_p)_o$	Peak deepwater wavelength
m	Model quantity
p	Prototype quantity
r	Subscript denoting ratio of model to prototype
S_a	Specific gravity of an individual armor unit relative to the water in which it is placed, $S_a = \gamma_a/\gamma_w$
T_p	Peak wave period
W_a	Weight of an individual armor unit, pcf
γ_a	Specific weight of an individual armor unit, pcf
γ_w	Specific weight of water, pcf
θ	Angle of structure slope measured from horizontal in degrees